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LOAD TESTS OF SOILS IN THE MIAMI AREA

by J. Calvin Jureit

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LOAD TESTS OF SOILS IN THE MIAMI AREA

J. Calvin Jureit, P.E., Jr. Mem A.S.C.E.
Engineer of Tests, Wingerter Laboratories, Inc., Miami, Florida

In the interest of selecting a bearing value for soils - how many pounds per square foot it will support - let's review the value of load tests and their interpretations.

As you know, load test results are reported to you by the laboratory in terms of settlement of a steel bearing plate (usually in 0.001 inches) for corresponding pressures applied by the plate (usually pounds per square foot). The procedure is to sustain pressure until settlement ceases and then apply another increment of pressure, and so on until that magic point called "failure" is reached - or until you reach the capacity of the equipment being used. Ordinarily, readings are also taken after the load is removed to determine the percent recovery - or how much the soil springs up again. Incidentally, these tests may be set up as dead loads on a platform acting through a column onto the bearing plate: or, as is common practice in the Miami area, the plate is jacked downward using for reaction a 16' steel truss, which is held to the ground by means of 4 or more screw anchors. Figure I shows the typical set-up.

All this sounds easy, but as with so many things, "there's many a slip twixt the cup and the lip". There is a technique to making these tests which is developed with experience. To mention a few details to be carefully observed:

1. Assuming the exact location has been chosen, consideration must be given to the depth at which the test is to be made and moisture condition of the soil during the test.

2. The soil under the test plate must be absolutely undisturbed by the digging operations in working down to that level. This takes care. In addition to density the soil may have a structure that can be damaged by careless preparation.

3. The soil is to be levelled carefully (very time consuming - especially with larger test plates) and covered with dry fine sand to fill any voids between the soil and the plate. When you think you've done this lift the plate vertically and fill the remaining voids which you are sure to find with more dry sand.

4. All plates should be thick enough to withstand bending or dishing. They must be stacked concentrically with the jack.

5. A spherical bearing block is necessary between the head of the jack and the truss to assure uniform pressure distribution (axial loading) through the plates. This is very important, since high pressures on one side of the plates will induce progressive failure.

6. At least 3 dials should be used for reading settlement of the bearing plates. These should be 120° apart and the same distance from the

center of the plates. In this way, any tilting action of the plates will be self-compensating. This is impossible to achieve with one dial. Two dials can be used, but they must be placed very exactly. For important results 3 or 4 dials is recommended.

7. Necessarily, the settlement dials are mounted from a cross-bar (referred to here as a reference bar), which is supported by stakes - well away from disturbance that could be caused by settlement of the plate or reaction from the anchors. Usually, the test site is subject to changing temperatures. This reference bar is very sensitive to such changes, so wood is recommended for this purpose.

8. The load is generally read from a hydraulic gage, mounted on the jack. These gages are not precision instruments, have considerable lag, and are subject sometimes to rough treatment - even though it be accidental. It is therefore necessary to calibrate these gages frequently against the gages of our standard compression testing machines.

9. First and last - the test needs to be conducted by competent personnel as the details above indicate.

So much for the problems of conducting the test itself, but let's get back to the value of the test to you, as an engineer. You've received a pressure-settlement curve from the laboratory and from it you determine allowable footing pressure.

In making this selection of allowable footing pressure it is suggested that you consider the following factors:

1. Choice of location of test. Generally, load tests should be preceded by borings to know whether the load test represents an average condition. Hand borings or penetration with a rod is better than nothing in making such a decision. Too often, we've been told, "Oh, make the test on the middle of the lot" -- and our crew has brought a softer location to attention by a few probings. Even visual means are of some value. Sometimes changes in vegetation on the site lead to soft areas.

2. The size of the plate used. The smaller the plate the less the settlement, generally speaking. Larger plates provide adjacent square feet of area and consequently their pressure zones, in the soil beneath, will overlap. This tends to extend the bulbs of effective pressures to lower levels and approach actual service. For this reason the writer employs the use of a 30" diameter plate (4.9 square feet), which is standard for Corps of Engineers Tests. This brings out an additional advantage for having a boring record of the site. If a soft strata exists, say 4 feet below the plate, a 1 square foot test plate will not send a very effective pressure into the soft layer.

3. The level at which the plate should be set. Generally it is requested that tests be made at the level of the bottom of the proposed footings. At this point it may be well to consider the value of load testing soft, low-lying areas before covering them with fill to determine the pressure they will take even though the pressure will eventually be conveyed through the fill. Knowing in advance this pressure value you have the opportunity to stabilize the soil if it tests low and you can determine the degree of compaction control required in placing the fill. Such a procedure may, in some cases, obviate the necessity for using piling.

An example of the value of compaction control was demonstrated at

a building site in Miami. Muck was stripped from the site, down to sand, which was under water. Rocky sand fill was dumped back in place of the muck. The water and working of the trucks helped somewhat with compaction and the surface was finally rolled in a helter-skelter fashion. The engineer had us make load tests and field density tests before permitting construction over this fill. Three load tests were made and numerous field density tests. The latter were used to "extend" the load test results quickly and economically. A correlation between the bearing values and degrees of compaction were found to be as follows:

Tests No.	% Compaction (Modified AASHO)	Load Test Result (p.s.f. at 0.1" settlement)
1	92.8%	1300
2	94.3%	2100
3	101.4%	2387

4. Rigidity of the Footing: Choice of an allowable pressure will depend too on the nature of the footing you place on the soil. A flexible-bottomed tank can stand the choice of a higher value than a rigid-framed structure that would suffer with slight differential settlements.

5. Order of Construction: Sometimes, some elements of construction may serve to "preload" the soil and permit the choice of a higher bearing value. An example of this type recently occurred in Miami. Four massive concrete footings for a water tower each tended to somewhat preload their respective sites. Any differential settlement in the footings prior to assembly of the tank on the footings did not affect the more or less rigid frame of the tank. In other words, a choice of bearing value based on differential settlement considerations alone could be added to the pressures caused by the weight of the footings themselves, - giving about a 10 percent total higher value than would have been permitted a structure requiring rigid support from the start.

6. The Test Report: Surely the report you are furnished reflects to some extent the degree of care and precision given to your load test. The pressure-settlement curve can be revealing in this respect also. Such a curve should be supplied with the report you receive. Nearly all curves should be fairly smooth and free of "wild points".

7. Choice of a Failure Load: Choosing the "failure" load from a curve can be a hair-pulling ordeal. There are numerous definitions of what constitutes failure. Usually, your local building code sets up some definition, such as, "allowable load shall be 1/2 of the load that produces 1/4" settlement after being sustained for 24 hours". Commonly, these codes fail to consider the size of plate, test level, rigidity of footing, etc.

The practice in Miami has been to employ a somewhat accelerated test procedure, since our soils are sandy and neutral stress will drain quickly under application of pressure. The procedure is essentially the same as the "Suggested Method of Test for Bearing Capacity of Subgrade Soils" by T. A. Middlebrooks, of the Office of the Chief of Engineers, War Department, Washington, D. C. - as given in ASTM's "Procedure for Testing Soils", July 1950 edition. The procedure has been to sustain each load increment for 2 minutes and if more than

0.001 inches of settlement occurs to sustain the same load for another time interval, squaring the time, or 4 minutes. Again, if settlement is more than 0.001 inches sustain the load for the square of the 3rd interval - or 9 minutes. This procedure continues until the average settlement of the plate is less than 0.001 inches in a time increment. Then, the load is increased and the same procedure is repeated. In general, the pressure that causes 1/10th inch settlement is designated as "failure" -- and 1/2 of that value allowed for construction. It is the writer's thought that this is too conservative -- especially for some loading conditions.

A frequent method of determining failure is by selection of the pressure at which an increased rate of settlement occurs - indicated by a characteristic "break" in the curve - and dividing by a safety factor of 2 for allowable working load.

It is seen that a load test has its complexities and that due consideration should be given to the many factors enumerated before selecting an allowable working load from a load-settlement curve.

Load Tests of Rock

And now, for those of you who will be interested in some test data. An interesting group of load tests made on Miami Oolite Rock have been assembled for your study. Copies of the findings are available for those who desire them. The results require almost individual study since there is too much scatter in them to assume any very definite patterns. As more tests are made they will add to this fund and perhaps in time a pattern will form.

For the benefit of those not familiar with Miami Oolite Rock, it is a porous rock, honeycombed with many cavities ranging from very small size up to 6 or 8 inches -- and generally pocked with vertical solution holes of almost one foot diameter. Usually these are filled with silica sand. The rock itself is high in calcium carbonate with silica making up nearly all of the balance.

Probably one reason that the results of tests on rock do not form a well defined pattern is that in all cases the rock has withstood a considerable load with very low corresponding settlements. The writer knows of no failure due to collapse of the rock - the only limitation has been capacity of the testing apparatus with loads up to 40,000 psf with maximum reliable settlements of 1/3", and that after sustaining the load for 2 hours. Most tests on rock settled much less under 40,000 psf loading -- generally in the order of 1/10". Heavy loads with low corresponding deflections call for super-precision if you expect to use the test results for correlation purposes -- as well as to just "get an answer" to get construction underway. In justice to the many who have made such tests, it is realized that the general pattern is to let these tests go until the last minute and then try to rush along the answer to "get the job started". Although a satisfactory answer can be obtained, it is seldom possible to adhere to the more precise practices under such pressure. And relaxing of precision leads to results that may not form patterns or plot into curves for us.

On the other hand, practices may be usually less precise in load testing most soils since fairly high settlements occur under rather low loads.

In 1948, the City of Miami Department of Water and Sewers found it necessary to determine the bearing power of Miami Oolite Limerock to construct their new Southwest Water Treatment Plant. They made an extensive effort to gather data on the bearing value of our local rock with little avail. All

they could find was 3 tests that had been made in 1946 in connection with the design of additions to Florida Power & Light Company's plant at Miami River, just east of N.W. 2nd Avenue. This is mentioned mostly for historic interest, because little detail was available to the writer except as referred to in the City of Miami report. What detail is given indicates comparatively inexact methods were used, such as measuring deflections with a taut wire against the loading column, and level shots made on 3 corners of the load platform. Also the tests were made 8 feet below grade, which may have been under water or at least in very wet condition, which makes precise methods difficult. At any rate, the record shows 5/8" settlement under 8 tons for the first test, which required 11 days to complete; 3/16" settlement under 8 tons for the second test, which was made in 4 days; and lastly, 1/16" for 7 tons in the third test - length of time not given.

The Water Department decided to have their own series of test conducted at their site. Their plan called for 2 series of tests, one at 2 feet below grade, the other at 5 feet depth. Each series included 3 tests, using circular bearing plates with areas of 1 square foot, 2 square feet and 3 square feet. The Water Department has been good enough to permit the writer to study the report of their tests and include the results in this paper. They are certainly to be commended for attacking this problem and for adding to the general knowledge. Their reports emphasize difficulties encountered in the first four tests with temperature changes affecting the elevation of their reference bar. This, in turn, affected settlement dial readings. Finally they adopted the use of 2 - 4" diameter pipes welded into a horizontal frame, mounted in grooves of cement grout to permit horizontal sliding of the pipes for relief of temperature stresses. Also, they covered the site with a 16' x 16' pyramidal army tent to protect the reference bar from a hot Florida sun. I'm sure the shade was welcomed by the men working in the pit also. They concluded that with all this effort their dial readings still were affected by .001" per 2° F. A 14° differential was experienced in the usual 24 hour test period. Their recommendation is to forget temperature effects if readings to the nearest .01" are satisfactory. Here again, that degree of precision is ample for merely determination of the safety of a location; but temperature corrections are necessary if inter-correlation of results is being seriously sought. This is apparent when you consider the fact that we have one test on record as not having deflected but little over half a hundredth of an inch under 40,000 psf., and several with merely .02" or .03" under similar loads.

In connection with this problem the writer intends to try the use of a glued-laminated reference bar, made from very dry wood, with the surfaces of the wood sealed against moisture by a coating such as shellac. This, with protection from the sun and sliding connections at the supporting stakes should improve harmful effects.

Not offered in the sense of criticism, but to acquaint you with a few additional factors that might have improved the Miami Water Department Tests for correlating purposes the following is suggested:

1. The use of 3 settlement dials instead of 2.
2. A ball-joint between the jack and the truss to assure as uniform loading as possible.
3. The use of cement grout between the rock and bearing plate. Loose, fine sand was used and could lead to different seating characteristics from one site to another.

4. Borings or test pits made directly under the test plate locations after all tests were completed would have been desirable especially since excavation during actual construction has revealed underground cavities -- one is about 15' square, horizontally, the cavity being about 1' thick, maximum -- an unusual occurrence.

With this background of prevailing conditions the summary of test results on rock as made by the Miami Water Department is as follows:

At 3 tons per square foot pressure, at a 2 foot depth:

.005" settlement using the 1 square foot plate
.008" " " " 2 " " "
.043" " " " 3 " " "

At 3 tons per square foot pressure at a 5 foot depth:

.0075" settlement using the 1 square foot plate
.029 " " " " 2 " " "
.031 " " " " 3 " " "

These figures verify that the size of the test area affects settlement. The rate is not the same in both series, but neither are the conditions the same. There is a difference in depths.

As noted before, none of the rock "failed" under ultimate loads but instead an anchor pulled or capacity of equipment acted as a limitation. Performance of the rock under ultimate loads, in the same order as given before, is as follows:

At 2 foot depth:

1 square foot plate, at 40,000 psf. settlement .198"
2 " " " " 20,000 psf. " .034"
3 " " " " 13,500 psf. " .076"

At 5 foot depth:

1 square foot plate, at 30,000 psf. settlement .078"
2 " " " " 20,000 psf. " .063"
3 " " " " 13,500 psf. " .062"

The latter is not subject to plotting since the ultimate loads are different, but is mentioned to give an idea of the performances of our rock under load.

The Water Department conducted several additional tests of considerable interest. In their repetitive test a 1 square foot plate at the 2 foot depth, immediately adjacent to the above tests was loaded to 10 tons, after 4 hours the plate had settled .031". Then the load was alternated between 7-1/2 tons and 10 tons at about 2 cycles of loading per hour. After 8 cycles no further settlement had occurred. The load was released to 500# and submitted to 7 cycles of load, alternating from 500# to 10 tons in another 8 hours. At the end of this loading final settlement was .037", which rebounded to .027" after removal of load.

In another test at the 2 foot depth, same location, the Water Department loaded a 1 square foot plate to 40,000 psf. In 2 hours settlement was .340 inches. At the end of 27 hours settlement was .346 inches, showing good stability. Note the much greater deflection here than in the other 40,000 psf tests nearby. They would likely be even more stable under a long sustained load.

Finally, there was conducted an accelerated test of the type in general use in Miami, similar to that described earlier in the paper. It was conducted

at the 5 foot depth, using a 1 square foot plate. Under 3 tons/sf. .009" settlement was noted and under ultimate load of 26,000 psf. a settlement of .116" obtained. The department's report states they felt this test was in good agreement with the other tests.

The writer feels sure you join in commendation of the Department of Water and Sewers of the City of Miami for undertaking such an extensive project, for making such valuable findings and for making them available to other engineers.

To the above fund of information I can add the results of 4 individual tests on Miami Oolite Limerock made by Wingerter Laboratories, Inc., with whom the writer is associated. Although made for 3 different clients they are, fortunately or unfortunately, within 5 blocks of one another - south of the Allapattah section of Miami.

The results are as follows:

Test No.	Depth	Plate Size	Settlement at 3 tons/sq. ft.	Ultimate Load p.s.f.	Maximum Settlement
1	6"	1 sq.ft. Circular	.00044"	41,250	.006"
2	6"	18" diam.	.020	15,000	.042"
3	9"	12" diam.	.0035"	40,000	.017"
4	12"	12" diam.	.0025"	24,500	.0145"

These tests are subject to some of the criticisms aforementioned if one were to use the results for over-all correlation purposes.

However, that was not the intent in any of the cases, but instead to determine a fair degree of safety in the foundation. To give you the background for these tests, however:

1. All were made with shade provided for the reference bar.
2. Reference bar was made of wood, 4" x 4".
3. Tests 1 & 2 employed 3 settlement dials. Earlier tests, 3 & 4, employed 2 dials.
4. In all cases a spherical bearing block was provided over the jack to prevent eccentric loading.
5. Fine sand was used as a cushion between the rock and the bearing plate.
6. As with the other tests, ultimate loads were limited only by an anchor pulling or capacity of equipment.
7. The test procedure was the accelerated method, described previously, same method as the last test described for the Water Department.

Load Tests of Soils

Wingerter Laboratories has conducted scores of load tests of soils in the past few years, mostly in the Miami area. The general procedure is as described in the early part of this paper. The writer worked up a tabulation of the results, but finds it impossible to make any cross-sectional correlations. It was his hope that curves could be developed for different soils, but the variables are far too numerous. Not only must different types of surface soils be considered but many other factors that have a great affect, are:

1. The different soil or soils which lie beneath the surface and the thick-

ness of each strata.

2. The degree of compaction of the soil of each strata.
3. Whether or not the material is a fill.
4. Location of the water table.
5. Moisture condition of the Surface soil at the time of testing.
6. Etc. etc.

An attempt to break down results on a geographic basis was also fruitless. Our soil is so variable that great variations in load test results are found on a series of tests on sand, for instance, that are all within the limited area of a few blocks.

Therefore I can offer only these generalized statements, which are also included with the copies of rock test results I mentioned before:

1. At 1/10th inch settlement the lowest value of any soil has been 400 psf. and the highest 12,200 psf. not counting tests on solid rock.
2. Soft, plastic, marly areas test from 400 psf. to possibly 1500 psf.
3. Marly fills have been found to test from 600 psf. to 3750 psf. on some of the drier marls that are mixed with sand and rock.
4. Silica sand - from 1500 to 5000 psf., with about 2500 psf. at normal compaction. And, incidentally, compacting the sand by vibrating or tamping can greatly improve results.
5. Rocky, sandy fill - from 1150 to 12,200 psf., with an average about 4000 psf. if fairly well compacted. The very high value of 12,200 psf. was obtained on a well compacted, rocky sandy fill, placed in 6 inch layers, wet and rolled.

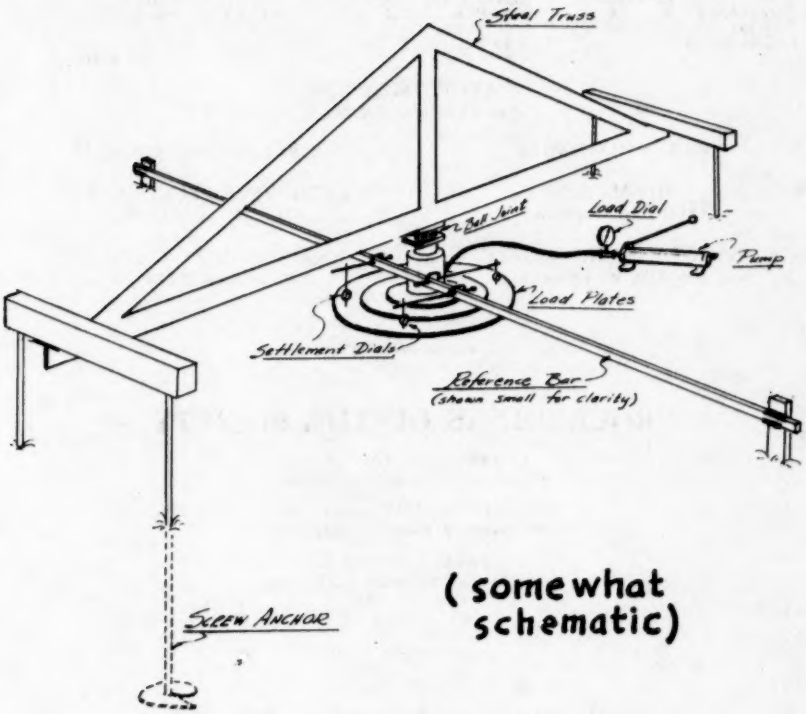
- 0 -

To those of you who will conduct load tests in the future, if you are interested in the carry-over value of your tests, the kind of results that can be correlated with other tests, whereby we can some day develop a series of bearing value curves - I urge you to do these four things:

1. Make a field density test of at least the upper strata of soil to fix the degree of compaction.
2. Augment your load test with a boring to rock, taken close by the loading site or prefereably under the plate after the test, and check the identification of the soil.
3. Keep a detailed record of the procedure and results.
4. And see that you have reasonable observance of the "fine points" as mentioned in this paper.

TYPICAL LOAD TEST SET-UP

as in use in the Miami Area....



(somewhat schematic)

FIG. I

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